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NRI INSTITUTE OF INFORMATION SCIENCE &TECHNOLOGY, BHOPAL

DEPARTMENT OF CIVIL ENGINEERING

SESSION 2014-2015

PROJECT REPORT ON

“DESIGN OF MULTISTORY BUILDING PROVIDING RESIDENCE FOR INDUSTRIAL AND COMMERCIAL PURPOSE”

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DECLARATION

We hereby declare that the work which is being presented in the project report entitled **“DESIGN OF MULTISTOREY BUILDING PROVIDING RESIDENCE FOR INDUSTRIAL AND COMMERCIAL PURPOSE”**
in the partial fulfillment of Bachelor of Engineering in Civil Engineering is an authentic record of our own work carried out under the guidance of
Prof. Sandeep K Shrivastava. The work has been carried out at NIIST, Bhopal.
The matter embodied in the report has not been submitted for the award of any other degree or diploma.

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BHOPAL

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CERTIFICATE

This is to certify that **Suraj Mishra, Suresh Chakrawarti, Nilesh Kumar Patel, Sraddhanand Meshram, Priyanshu Suryawanshi,** students of Fourth year (VII semester) Bachelor of Civil Engineering, NIIST have successfully completed their Major Project Report on **"Design of Multistory Building"**.

We approve the project for the submission for the partial fulfillment of the requirement for the award of degree in Civil Engineering.

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We are also grateful to respected **Mr. J.P. Nanda,** HOD (Department of Civil Engineering) NIIST, Bhopal for permitting us to utilize all the necessary facilities of the institution.

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Thank You

Table of Contents

Sr. No.	Topic	Page no.
1.	Introduction i). Effective Span ii). Stiffness iii). Loads iv). Analysis	6
2.	Load Distribution	11
3.	Moment Calculation by KANI’S Method	14
4.	Design of One way Slab	19
5.	Design of Two way Slab	25
6.	Design of T-Beam	32
7.	Design of Column	37
8.	Design of Staircase	40
9.	Design of Flat Footing	46
10.	Conclusion	51
11.	References	52

Multistory Building

1.1. INTRODUCTION

The aim of this project is to design a Multistory Building (G+2) for residential purpose, taking earthquake load into consideration.
Multistory buildings are very commonly seen in cities.
Construction of such tall buildings are possible only by going to a set of rigidly interconnected beams and column. These rigidly interconnected beams and columns of multi bay and multistoried are called Buildings frames.
To avoid long distance of travel, cities are growing vertically rather than horizontally. In other words multistory buildings are preferred in cities.
Building laws of many cities permits construction of ground plus three storey buildings without lifts.

The loads from walls and beams are transformed to beams, rotation of beams take place. Since, beams are rigidly connected to column, the rotation of column also take place. Thus any load applied any where on beam is shared by entire network of beam and columns.

1.2. EFFECTIVE SPAN

As per IS 456-2000, in the analysis of frames, the effective length of members shall be center to centre distance (clause 22.2 d)

1.3. STIFFNESS

For the analysis of frame, the relative stiffness values of various members are required. IS 456-2000 clause suggests the relative stiffness of the members may be based on the moment of inertia of the section.
The made shall be consistent for all the members of the structure throughout analysis. It needs arriving at member sizes before designing. The sizes are selected on the basis of architectural, economic and structural considerations.
For **Beams** span to depth ratio preferred is 12 to 15. Width is kept (1/3) to (1/2) of depth, but some times they are fixed on architectural consideration.
Column sizes are to be selected on the basis of experience.

It is to be noted that in **Multistory frames,** columns of upper stories carry less axial force but more moments, while columns of lower storey carry more axial loads and less moments.
Design can roughly estimate the axial load on lower storey column and arrive at sizes of the column.
Next two to three stories can have same size. Beyond that, sizes may be reduced. Stiffness of member is given by (I/L).

1.4. LOADS

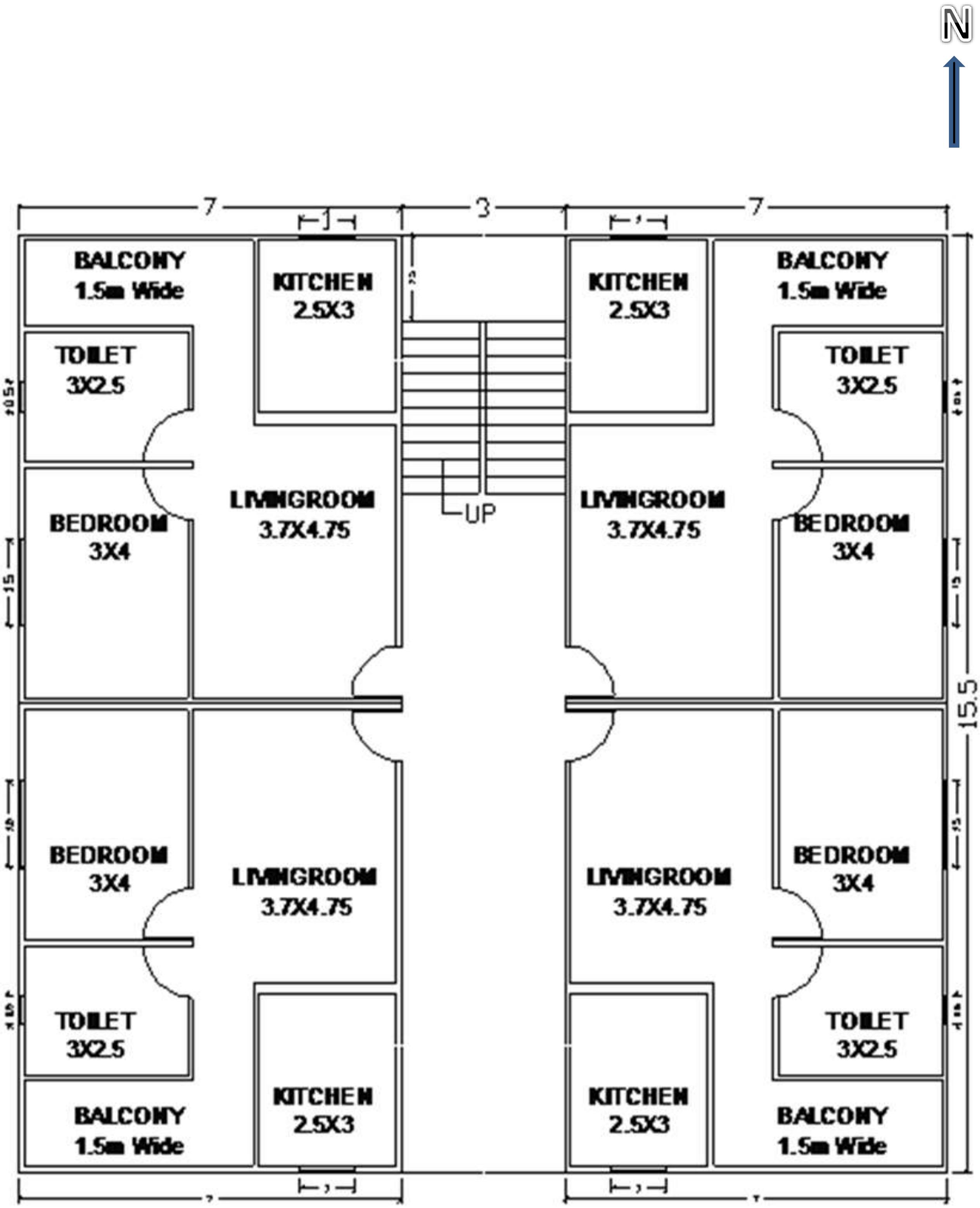
For Multistory frames Dead load, imposed load (live load), wind load and earthquake loads are important for designing.
The IS code suggests following load combination to get designed loads:

1. 1.5DL + 1.5IL
2. 1.5DL + 1.5WL
3. 1.5DL + 1.5EL
4. 1.2DL + IL + 1.2WL

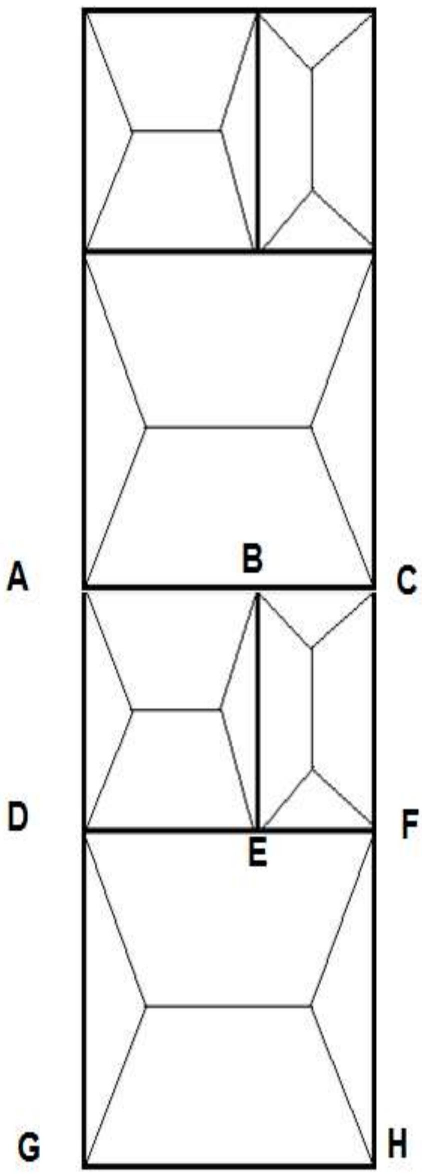
✓ 5. 1.2DL + IL + 1.2EL

1.5 ANALYSIS

It may be analyzed as a set of intersecting frames taking care of loads from **triangular pattern of loads** from floors. However, IS 456-2000 (Clause 22.42) permits the analysis of frames by approximate methods like:
Portal method, cantilever method, Substitute frame method for Dead loads, factor method for wind loads; to arrive at design moments, shear and other forces.
We have adopted **KANI'S** method for frame analysis.



TYPICAL FLOOR PLAN



Load due to slab: (KN)

- ✓A = 13.79+13.79+10.575+10.575 = 48.73
  B = 13.76+13.79 = 27.58
  C = A = 48.73
- ✓D = 13.79+10.575+8.44+4.22 = 37.025
  E = 13.79+8.44+4.22+4.11+2.93 = 33.49
  F = 13.79+10.575+2.93+4.11 = 31.405
- ✓G = 4.22+8.44 = 12.66
  H = 4.22+8.44+4.11+2.93 = 19.70
  I = 4.11+2.93 = 7.07

Load due to slab: (KN)

- ✓A = 2.25
  B = 2.25+3 = 5.25
  C = 3
- ✓D = 3.375
  E = 3.375+1.875 = 5.25
  F = 1.875
- ✓G =3.375
  H =6.675
  I =3.375

Fig1. Triangular Pattern of load distribution.

Loadings on Frame:

From Top- 1) = (48.73+2.25) + 2(37.025+3.375) + 2(12.66+3.375)

= 163.85 KN

UDL = (163.85/15.5) = 10.57 + 1.5(i.e. LL)

= **13 KN/m**

2) => 13 + {[(0.2X0.3X3.3X25)X5]/15.5} + 13

= **28KN/m**

3) => 13 + 28 + (13 + 1.6)

= **56KN/m**

4) => 13 + 28 + 56 + (13 + 1.6)

= **112KN/m**

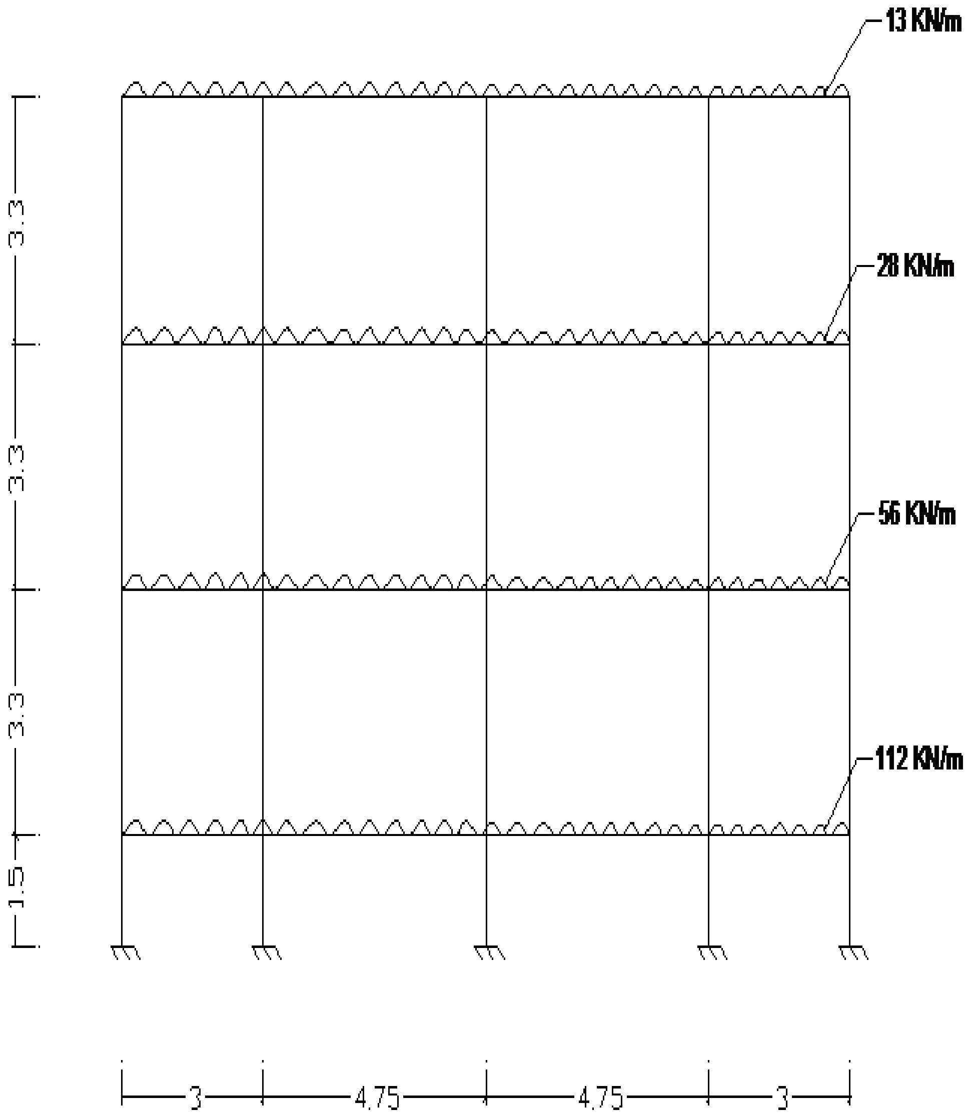


Fig. LOADED FRAME

KANI'S METHOD

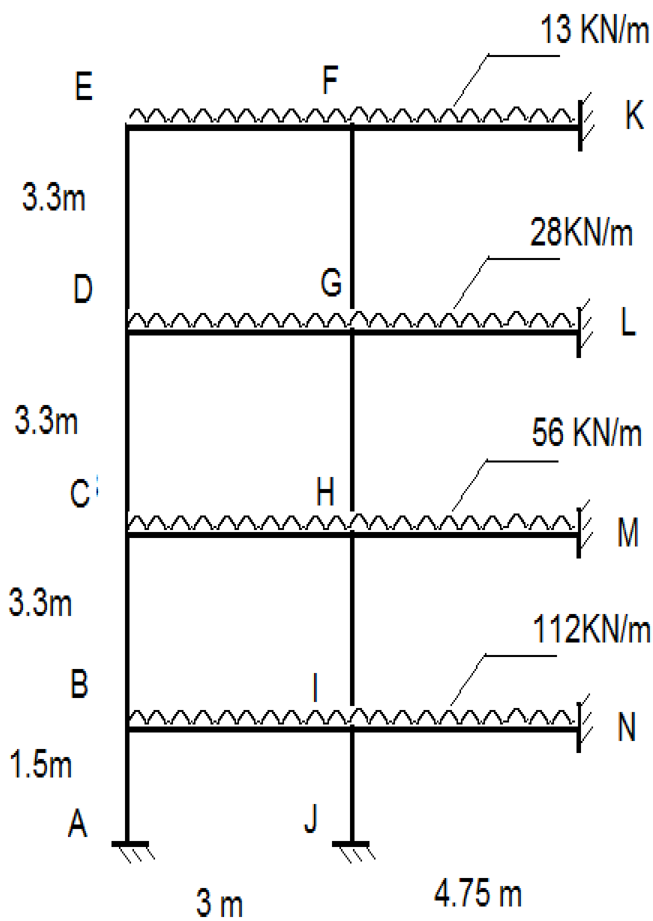


Fig 2. Substitute Frame
(Line of symmetry passes through column)

Fixed End moments: (KMm)

$$Mf_{BI} = -Mf_{IB} = \frac{wl^2}{8} = \frac{112 \times 3^2}{8} = 84$$

$$Mf_{IN} = -Mf_{NI} = 210.58$$

$$Mf_{CH} = -Mf_{HC} = 42$$

$$Mf_{DG} = -Mf_{GD} = 21$$

$$Mf_{EF} = -Mf_{FE} = 9.75$$

$$Mf_{HM} = -Mf_{MH} = 105.3$$

$$Mf_{GL} = -Mf_{LG} = 52.65$$

$$Mf_{FK} = -Mf_{KF} = 24.44$$

Moment of Inertia:

$$I = \frac{bd^3}{12} = \frac{0.2 \times 0.3^3}{12} = 2 \times 10^{-4} m^4$$

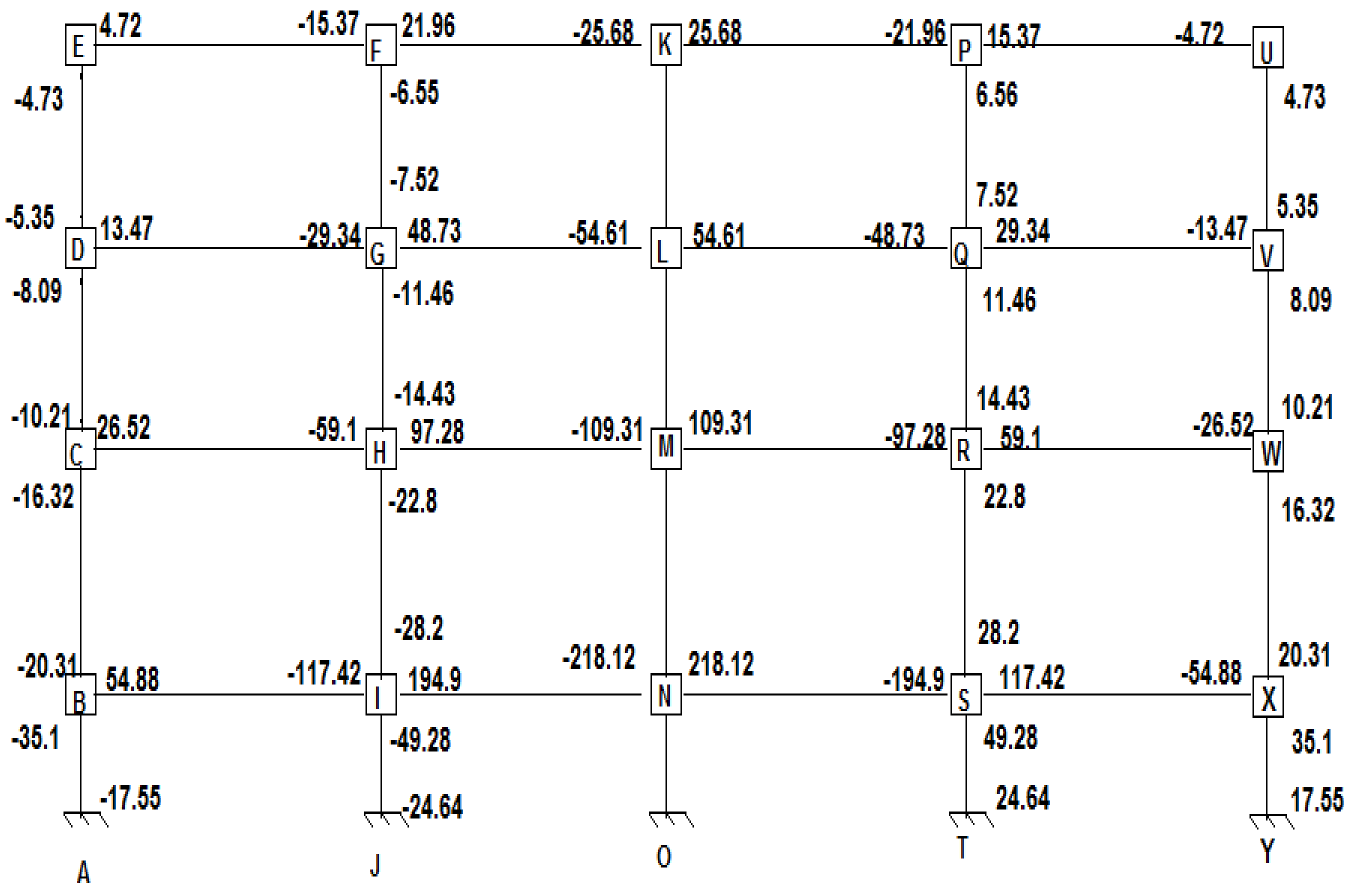
Rotation Factors:

Joint	Member	Stiffness	R.Stiffness	R.F.
	BA	2I/1.5		-0.26
B	BI	2I/3	86I/33	-0.12
	BC	2I/3.3		-0.12
	CB	2I/3.3		-0.16
C	CH	2I/3	62I/33	-0.18
	CD	2I/3.3		-0.16
	DC	2I/3.3		-0.16
D	DG	2I/3	62I/33	-0.18
	DE	2I/3.3		-0.16
E	ED	2I/3.3	14I/11	-0.24
	EF	2I/3		-0.26
	IJ	2I1.5		-0.22
I	IN	2I/4.75	1898I/627	-0.07
	IH	2I/3.3		-0.10
	IB	2I/3		-0.11

Rotation Factors:

Joint	Member	Stiffness	R.Stiffness	R.F.
	HI	2I/3.3		-0.13
H	HM	2I/4.75	1442I/627	-0.10
	HG	2I/3.3		-0.13
	HC	2I/3		-0.14
	GH	2I/3.3		-0.13
G	GL	2I/4.75	1442I/627	-0.10
	GF	2I/3.3		-0.13
	GD	2I/3		-0.14
	FG	2I/3.3		-0.18
F	FK	2I/4.75	354I/206	-0.12
	FE	2I/3		-0.20

Final – End Moments (KNm)



Check $\sum M = 0$ at each joint. Taking moments due to earthquake load = 6KNm Remark : Checked **OK.**

Design of One way Slab

- Data given, (m)

  Clear span (or Room size) = 7mX3m

  L.L = 1.5 KN/m^2 , support thickness = 200mm

  Surface finishing = 1 KN/m^2

  Using M20 & Fe 415

Step 1 :- Design constant for M20 concrete & Fe415 steel

Fck =20 N/mm^2 , Fy = 415 N/mm^2

Mu_{limit} = 0.138 fck bd^2

Xu = 0.479 d

Step 2 :- Type of Slab- ly/lx = 7/3 = 2.33 > 2

therefore design One way slab,

considering shorter span.

Step 3 :- Effective depth of span

for continuous slab one way

deff = l/(26 X M.F)

assume Modification factor

M.F =1.3 (IS456:2000 Page - 38)

= 3000/(26 X 1.3)

provide depth = 88.75 ≈ 90 mm ,

Take d_{eff} = 125 mm

Overall depth D = d +(c.c+Ø/2) assume dia. of bar 10mm

= 125 +(20+10/2) c.c= 20mm

= 125+25 =150 mm

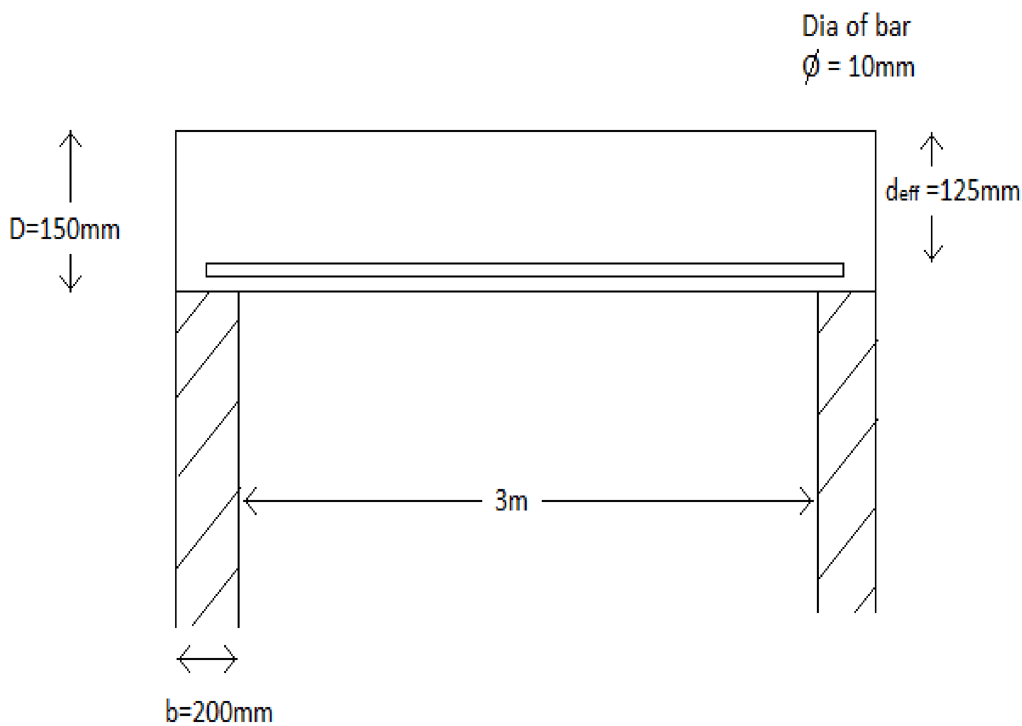


Fig. Diagrammatic Representation

Step 4 :- Effective Span (l_{eff})-

(1) L+ b = 3000 + 200 = 3200 mm

(2) L+ b = 3000 + 125 = 3125 mm (which ever is less)

thus l_{eff} = 3.125 m

Step 5 :- Load Calculation-

(1) Dead load of slab = 1x1x(d/1000) ρ_{rcc}

= (150/1000)x25 =3.75KN/m^2

(2) Live load = 1.5 KN/m^2

(3) Finishing load = 1 KN/m^2

Working load w = 6.25 KN/m^2

Factored load w_u = 1.5w = 1.5x6.25

= 9.375 KN/m^2

Step 6 :- Factored Bending Moment (M_u)-

Mu = coeff. x w_u x l_{eff}^2

From :

IS 456:2000

Page 36 Tabel no.12

[BM coefficients of Continuous slab at the mid of interior span for dead load & imposed load (fixed) + 1/16]

Mu = (9.375x3.125^2)/16

Mu = 5.722 KNm per meter width of slab

Step 7 :- Check for depth ($d_{req.}$)-

Effective depth required $d_{req.}$ = $\sqrt{(Mu/0.138fck\ b}$

= $\sqrt{(5.722 x 10^6)/(0.138 x 20 x 1000)}$

$d_{req.}$ = 45.53 mm

$d_{req.}$ < $d_{provided}$

OK-SAFE

Step 8 :- Main Steel –

A_{st} = $0.5\ f_{ck}/f_y\ [\ 1-\sqrt{1-(4.6\ Mu/f_{ck}\ bd^2)}\ ]\ bd$

A_{st} = $0.5x20/415[1- \sqrt{1-(4.6 \times 5.722 \times 10^6 / 20 \times 1000 \times 125^2)}]\ 1000 \times 125$

A_{st} = 129.638 ≈ 130 mm²

and A_{stmin} = 0.0012 bD

= 0.0012 x 1000 x 150

= 180 mm²

here, A_{stmin} > A_{st}

therefore use A_{stmin} i.e. 180 mm²

Step 9 :- Spacing Of Main Bar –

(1) $(1000 \times A_{st}) / A_{stmin} = (1000 \times \pi/4 \times 10^2) / 180$

$= 437$ mm

(2) $3d = 3 \times 125 = 375$ mm

(3) 300 mm = 300 mm

(which ever is less)

provide (δ = 300 mm)

Ø = 10 mm @ 300 mm c/c spacing along shorter span.

Length of rod = 3000 – (2 x clear cover)

= 3000 – (2 x 20) = 2960 mm

provide 10 Ø @ 200 mm c/c & extra at top upto l/4 i.e. 0.8 m both supports

Step 10 :- Spacing Of Distribution steel –

here $A_{stmin} = 180\ mm^2$

(assuming dia. Of bar 8 mm)

(1) $(1000 \times \pi/4 \times 8^2)/180 = 279.25 \approx 280$ mm

(2) $5d = 5 \times 125 = 625$ mm

(3) 450 mm

(which ever is less)

provide 8 mm dia. Of distribution bar @ 280 mm c/c spacing across main bar

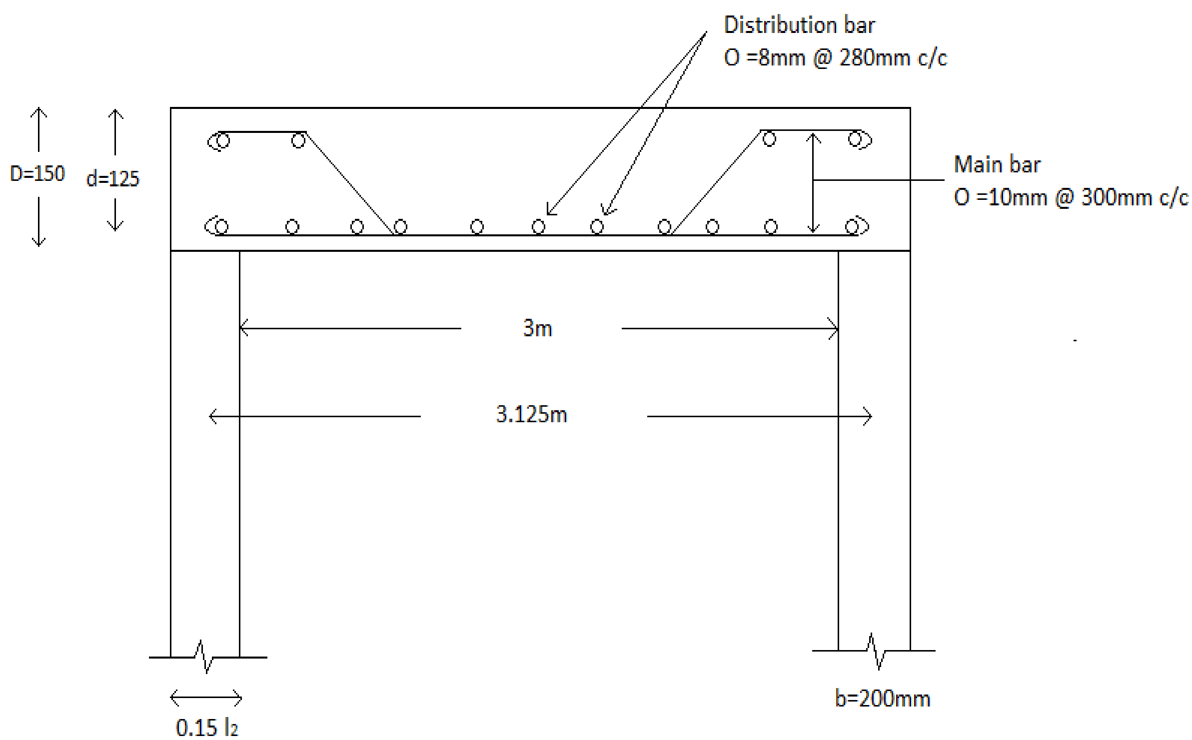


Fig. Reinforcement Details in One way Slab.

Design of Two way Slab

Given Data-

Size of slab (m) = 7 x 4.75

Live load = 2 KN/m²

support thickness = 200 mm

Finishing = 1 KN/m²

Use M20 & Fe415

Step 1):- Design constant-

f_{ck} = 20 MPa, f_y = 415 MPa

$M_{u\,lim} = 0.138\, f_{ck}\, bd^2$

$X_u = 0.479\, d$

Step 2):- Type of Slab-

$l_y/l_x = 7/4.75 = 1.5 < 2$

(Two way slab)

Step 3):- $D_{effx} = l_x/26 \times 1.5 = 4750/26 \times 1.5 = 121.7$ mm $= d_x$

≈ 125 mm $= d_x$

Assume 10 Ø, clear cover 20 mm

$d_y = 125 - 10 = 115$ mm

Overall depth of slab D = d+(c.c.+ Ø/2)

D = 125 + 20 + 5 = 150 mm

Step 4):- Effective length of Slab

here support thickness = 200 mm

Shorter Span	Longer Span
i). Clear span + d_x 4750 + 125=4875mm	i). Clear span + d_y 7000 + 115=7115mm
ii). Clr span + support width 4750 + 200=4950mm	ii). Clear span + b 7000 + 200=7200mm

(which ever is less)

l_x = 4.875 m l_y = 7.115 m

Step 5):- Load-

i). D.L. = 1x1x150/1000x25 = 3.75KN/m²

ii). Live load = 2KN/m²

iii). Finishing = 1KN/m²

Working load = 6.75KN/m²

W_u = 1.5 x 6.75 = 10.125KN/m²

Step 6):- Moments-

l_y/l_x = 7.115/4.875 = 1.46

Moment coefficients:

l_y/l_x	α_x	α_y
1.4	0.099	0.051
<u>1.46</u>		
1.5	0.104	0.046

α_x = 0.099 + (0.104-0.099)/(1.5-1.4) x (1.46-1.4)
= 0.102
α_y = 0.051 + (0.046-0.05)/(1.5-1.4) x (1.46-1.4)
= 0.048

$M_x = \alpha_x W_u l_x^2 = 0.102x10.125x4.875^2 = 24.54$ KNm
$M_y = \alpha_y W_u l_y^2 = 0.048x10.125x4.875^2 = 11.55$ KNm

Step 7:-Check for depth-

$d_{required} = \sqrt{}(M_x/0.138x20x1000)$
$= \sqrt{}[(24.54x10^6)/(0.138x20x1000)]$
= 94.29 mm ≈ 95 mm

$d_{req} < d_{provided}$

OK SAFE.

Step 8:- Area of Main Steel-

A_{stx} = $0.5(f_{ck}/f_y)\left[1- \sqrt{1-\{(4.6 \times M_x)/(f_{ck}\, b\, d^2x)\}}\right] b\, d_x$

$= 0.5(20/415)\left[1- \sqrt{1-\{(4.6 \times 24.54 \times 10^6)/(20 \times 1000 \times 125^2)\}}\right] 1000 \times 125$

$= 604.72\ mm^2$

$A_{sty} = 0.5(20/415)\left[1- \sqrt{1-\{(4.6 \times 11.55 \times 10^6)/(20 \times 1000 \times 115^2)\}}\right] 1000 \times 11.5$

$= 293.89\ mm^2$

$A_{stmin} = (0.0012 \times bD) = (0.0012 \times 1000 \times 150)$

$= 180\ mm^2$

A_{stx} & A_{sty} > A_{stmin}

Hence, use A_{stx} & A_{sty} .

Step 10:- Spacing of main bar -

assume dia. of main bar Ø = 10 mm

Shorter span	Long span
(1) 1000 X π/4 X 10^2/ A_{stx} = 129.88 ≈ 120 mm	(1) 1000 X π/4 X 10^2/ 293.89 = 267.24 ≈ 260 mm
(2) $3d_x$ = 3 X 12 = 375	(2) $3d_y$ = 3 X 115 = 345
(3) 300 mm	(3) 300 mm

(which ever is less)

provide 10 Ø @ 120 c/c

provide 10 @ 260 c/c

(3/4 l) span middle strip

Step 11:- Distribution Steel -

$A_{stmin} = 180\ mm^2$

spacing assume $\emptyset = 8$ mm

(1) $1000 \times \pi/4 \times 8^2/180 = 279.25$ mm

(2) $5d_x = 5 \times 125 = 625$

$5d_y = 5 \times 115 = 575$

(3) 450 mm

provide 8 Ø @ 270 c/c edge strip (span/ 8)

Step 12:- Check for deflection –

$d_{provided} = l/(26 \times MF)$

$A_{stprovided} = (1000 \times \pi/4 \times 10^2)/120$

$= 654.5\ mm^2$

$A_{strequired} = 604.72\ mm^2$

% of steel = $A_{stprovided} /(b \times d \times 1000)$

$= 0.37\ \%$

$F_5 = 0.58 \times f_y A_{strequired}/ A_{stprovided}$

$F_5 = 222.4$ IS 456 : 2000

MF = 1.5

$d_{required}$ 121.8 mm

$d_{provided}$ 125 mm

$d_{required} < d_{provided}$

OK-SAFE

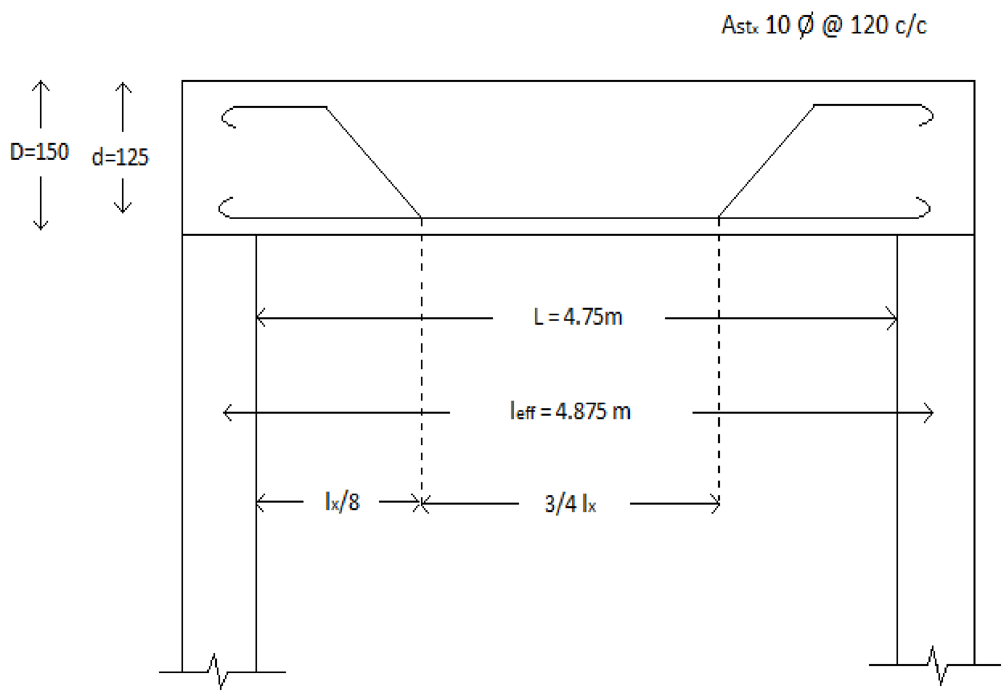


Fig. Reinforcement Details in Two way Slab.

Design of T- BEAM

Data :

Clear span(L) = 4.75 m, f_{ck} = 20 N/mm^2

Depth of flange (D_f) = 150 mm, f_y = 415 N/mm^2

Depth of web (b_w) = 200 mm

Imposed Load = 112 KN/m,

Step-1 Effective Depth (d):

$$d_{eff} = \left(\frac{span}{15}\right) = \left(\frac{4750}{15}\right) = 316.67mm \approx 320mm$$

Adopt D = 320 + 20 + 25 = 360 mm

Step-2 Effective Span (l_{eff}):

The least of

(i) Centre to centre of support = 4.75+0.2 = 4.95 m

(ii) Clear span + effective depth = 4.75 + 0.32 = 5.1 m

$\therefore$ Effective span = 4.95 m

Step-3 Loads

Imposed load = 112 KN/m

Ultimate load = 1.5 X 112 = 168 KN/m

Step-4 Ultimate BM and Shear force

$$M_u = \frac{wl^2}{8} = 0.125 \times 168 \times 4.95^2 = 514.55KN - m$$

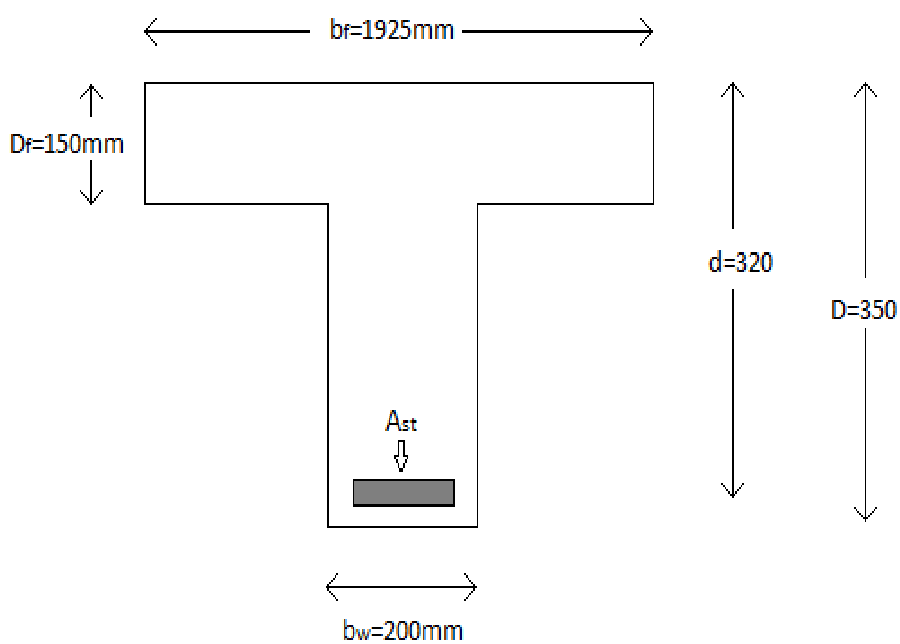
$$V_u = \frac{wl^2}{2} = 0.5 \times 168 \times 4.95 = 415.8KN$$

Step-5 Effective width of flange(b_f):

$$b_f = \left(\frac{l}{6} + b_w + 6D_f \right)$$

=[(4.95/6)+0.2+(6X0.15)

= 1925 mm



Step-6 Moment capacity of Flange section(M_{uf}):-

$M_{uf} = b_f\, D_f\, 0.36 f_{ck}\, (d - 0.416 D_f)$

=1925 X 150 X 0.36 X 20 X (320 – 0.416X150)

= 535.55 KN-m

Since, $M_u < M_{uf}$ i.e. Neutral axis is within the Flange, Hence, the section is treated as Rectangular with $b = b_f$ for designing reinforcement.

Step-7 Tension Reinforcements:-

$$M_u = (.87 f_y A_{st} d)\left[1 - \frac{A_{st} f_y}{b d f_{ck}}\right]$$

$514.55 \times 10^6 = A_{st} \times 0.87 \times 415 \times 320 \{1 - (A_{st} \times 415)/(1925 \times 320 \times 20)\}$

A_{st} = 545.651 mm^2

A_{st} = 545.651 mm^2

Provide 3 nos. 14Ø at bottom,

2 nos. 10Ø at top, & provide (l/4) extra at top

total A_{st} = 618.89 mm^2

Step-8 Shear Reinforcement:-

$\tau_v = (V_u / b_w\, d) = 415.8X10^3/(200X320)$

$= 6.49\ N/mm^2$

$P_t = 100\ A_{st} / b_w d \quad = \ 100X545.651/(200X320)$

$= 0.853\ m$

from IS 456:2000, page no.73,table-19,

Design shear strength of concrete (M20)

$\tau_c = 0.28\ N/mm^2$

Balance Shear => $V_{us} = [V_{us} – (\tau_c\, bd)]$

$V_{us} = [415.8 – (0.28X200X320\,)10^{-3}]$

$= 397.88\ KN$

Using 8 mm dia, 2 legged stirupps,

Spacing is given by,

$S_V = (0.87 f_y A_{sv}\ d/V_{us})$

$S_V = (0.87X415\ X(\pi \div 4)X8^2/397.88X10^3)$

$S_v = 220\ mm \approx 200\ mm$

provide spacing of 100 mm and gradually increase to 200 mm at centre of span

Step9:- Check for deflection Control –

$P_t = 100\ A_{st}/(b_f\ d)$

$= (100 \times 5378)/(2025 \times 320) = 0.83$

$b_w/b_f = 200/2025 = 0.099$

$(L/d)_{provided} = L/d \times K_t \times K_c \times K_f$

$4950/320 = 20 \times 1.05 \times 1 \times 0.94$

$15.46 < 19.74$

hence, check for deflection is satisfactory.

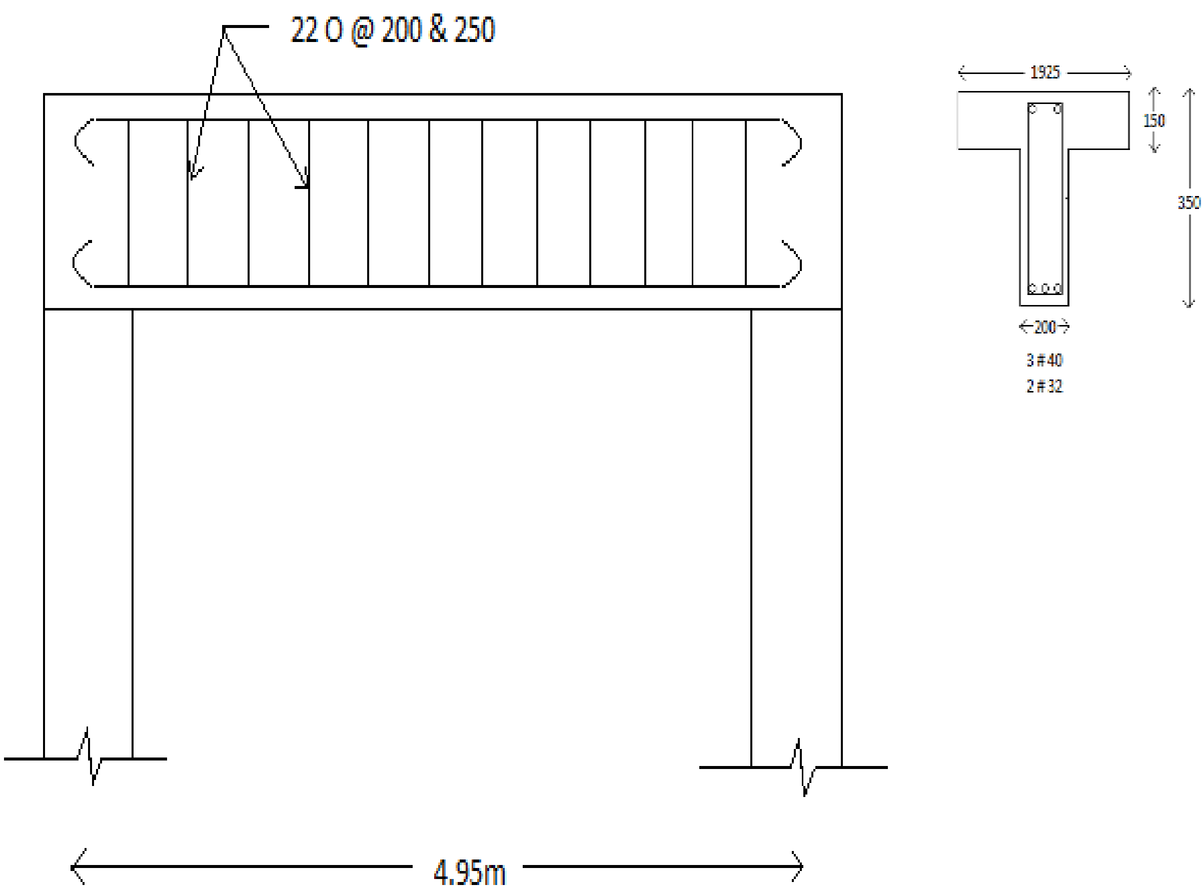


Fig. Reinforcement Details in T-beam.

Design of Column

Data-

Axial load on column = 400 KN3737

Length (L) = 3.3 KN

Column size = 200X300

Adopt M20 and Fe415

F_{ck} = 20 N/mm² F_y = 415N/mm²

Step1:- Effective length of column-

both end fixed l = 0.6 L

= 0.65 X 3.3 = 2.145 m

factored load Pu = 1.5 X 400 = 600 KN

Step2:-Slenderness ratio-

unsupported length/least lateral dimension

{L_{eff}/D} = 2145/200 =10.725 < 12

hence column is designed as short column

Step3:-Minimum Eccentricity-

$e_{min} = \left[(l/500)+(D/30)\right]$ or 20 mm

= 10.96 mm or 20 mm

e_{min} = 20 mm

Check,

10.96/200 = 0.05 ≤ 0.05

OK

Hence, codal formula for short column is applicable.

Step4:- Main steel (Longitudinal reinforcement)-

$P_u = [(0.4 X f_{ck} Ac) + (0.67 F_y Asc)]$

Ac = area of concrete

Asc = area of steel

Ag = gross area (200x300 = 60000 mm^2)

$600X10^3 = 0.4X20X0.99Ag + 0.67X415X0.01Ag$

Ag = 56072.15 mm^2

Asc = 0.01 Ag = 561 mm^2

Asc_{min} = 0.08 Ag = 448.57 mm^2 ≈ 449 mm^2

provide 12Ø - 6Nos(Total Area of steel = 678.58 mm^2)

Step5:- Design of Lateral Ties-

(1) Dia. of ties $Ø_{tie}$ = Øtie / 4 =12/4 = 3 mm

Øtie = 8 mm (for Fe 415)

Spacing-

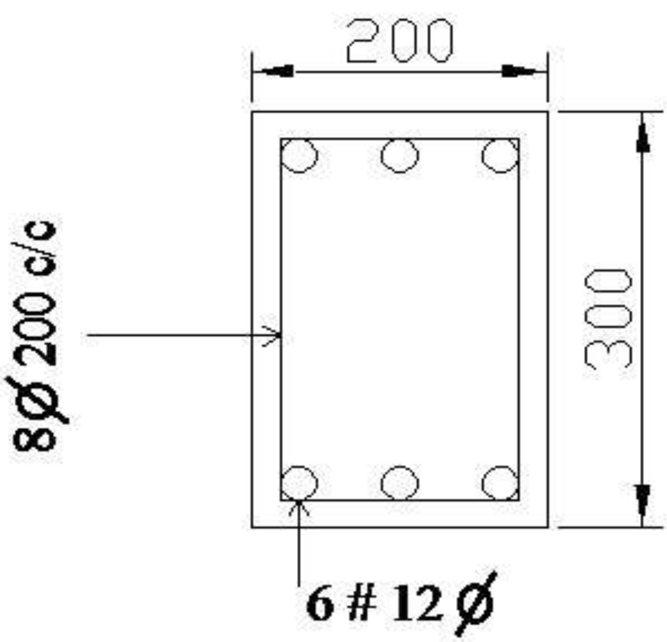
a) least lateral dimension = 200 mm

b) 16 X $Ø_{main}$ = 16X12= 192 mm

c) 300 mm

which ever is less

provide 8 Ø @ 200c/c



Design of Stair case (Dog legged)

Data,

ht. Of storey = 3.3 m

size of stair hall =4.5mX3m

L.L = 2 KN/m^2

supported width = 200 mm

Step 1 :- Design constants –

using M20 and fe415

F_{ck} = 20 Mpa

F_y = 415 Mpa

$M_{ulimit} = 0.138\ F_{ck} bd^2$

Step 2 :- Arrangement of stair-

Ht. Of storey = 3.3 m

Ht. Of flight = 3.3/2 = 1.65 m

assume R = 150 mm , T = 300 mm

No. Or riser = 1650/150 = 11

No. Of tread = 11-1 = 10

Going G = no. Of tread X T

= 10 X 300 = 3000 mm

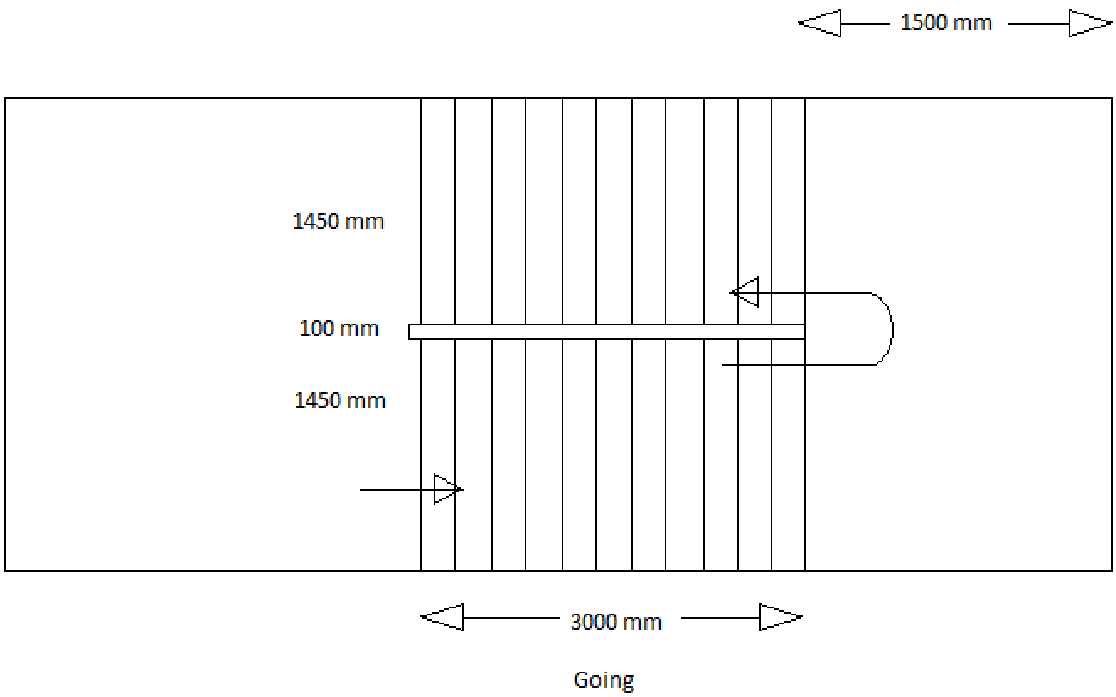


Fig. Arrangement of Steps in Staircase.

Step 3 :- Effective length-

l_{eff} = c/c dist. b/w support

= 3000 + 1500 +200/2 = 4600 mm

Step 4 :- Effective depth of waist slab –

$d \approx l/25 = 4600/25 = 184 \approx 180$

assume 10 Ø and clear cover 15 mm

D = 180 + (15+10/2) = 200 mm

but we adopted D = 150 mm

Step 5 :- Load calculation (unit area) –

(1) Self wt. Of waist slab in horizontal area

$= w_s \times \sqrt{(R^2+T^2)}/T$

$= (1X1XD/1000)\rho_{rcc} \times \sqrt{(150^2+300^2)}/300$

$= 4.19\ KN/m^2$

(2) Self wt. Of step per meter length

$= (R/2)\rho_{pcc} = (150/2)24 = 1.8$ KN/m²

(3) Finishing load minimum = 0.75 KN/m²

(4) L.L = 2 KN/m²

$w = 8.74$

$w_u = 1.5\,w = 13.11$ KN/m²

Step 6:- Bending moment –

$M_u = wl^2/8 = (13.11 \times 4.6^2)/8 = 34.67$ KN/m

Step 7:- Check for effective depth –

$d_{required} = \sqrt{(M_u/0.138 f_{ck} b)}$

$= \sqrt{(34.67 \times 10^6 / 0.138 \times 20 \times 1000)}$

$d_{required} = 112.078$ mm

$d_{required} < d_{provided}$ (i.e.= 150)

OK SAFE

Step 8:- Main steel –

$A_{st} = 0.5X20/415[1-\sqrt{1-\{(4.6X34.67X10^6)/(20X1000X150^2)\}}]$

$\approx 711\ mm^2$

$A_{stmin} = 0.0012X1000X150 = 180\ mm^2$

use $A_{st} = 711\ mm^2$

Step 9:- Spacing of Main bar-

(1) $(1000X\pi/4X10^2)/711$ assume 10Ø

=110.46 mm

(2) 3X150

(3) 300mm

which ever is less

Main bar provide 10Ø @ 100 c/c

Step 10:- Distribution bar-

use A_{stmin} = 180 assume Ø = 8 mm

(1) $(1000X\pi/4X8^2)/180$ = 279.15 mm

(2) 5D = 5X150 =750 mm

(3) 450 mm

distribution bar provide 8Ø @ 250 c/c spacing

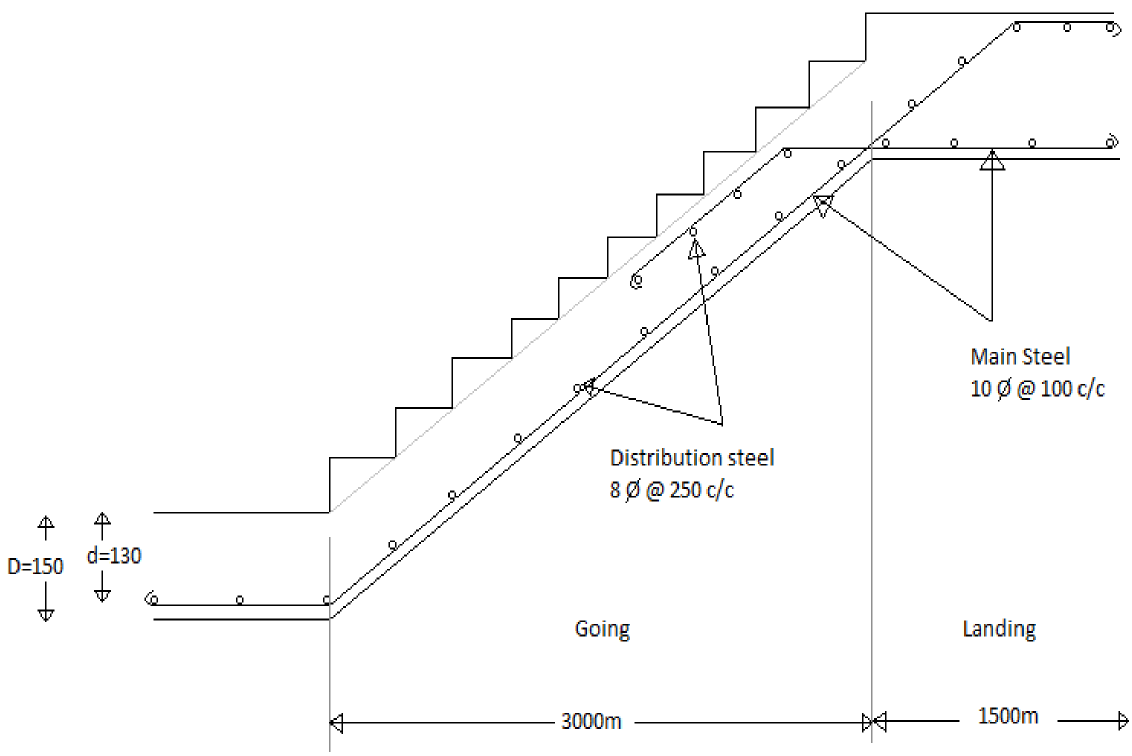


Fig. Reinforcement Details in Stairs

Design of Flat Footing

Data:

Assume SBC of soil = 200 KN/m²
Reinforcement concrete column size = 200 X 300
Axial service load P = 400 KN
Adopt M20 & Fe415

Step 1: Calculation of Load-

a) Load on column = 400KN
b) Self wt. of footing = 10% of column
= 400 X (10/100) = 40 KN

Total load = 440 KN

Factored load Wu = 1.5 X 440 = 660 KN

Step 2: Area of footing-

$$= \frac{Load(withoutfactor)}{SBCofSoil} = \frac{440}{200} = 2.2m^2$$

Assuming square footing,
Size of footing = $\sqrt{2.2} = 1.45m$
Adopt size of footing = 1.5m X 1.5m

Step 3: Net upward pressure-

$$Pn_u = \frac{FactoredLoad}{actualAreaofFooting} = \frac{660}{1.5X1.5} = 293.33KN/m^2$$

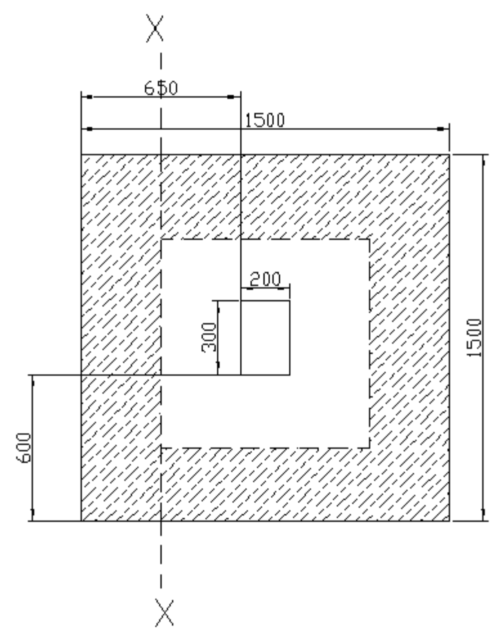
Step 4: Bending Moment calculation-

Maximum bending moment will be on the face of column,

M = F X Distance of C.G.

= (area X stress) x (0.65/2)

= 92.95 KNm



Step 5: Depth of Footing –

$$d_{required} = \sqrt{\frac{M}{0.138 f_{ck} b}}$$

$$d_{required} = \sqrt{\frac{92.95X10^6}{0.138X20X200}} = 410.35mm \Rightarrow Adopt \approx 420mm$$

Assume cover = 60mm
Thus, Overall Depth = 420+60 = 480mm

Step 6: Main Steel calculation-

$$A_{st} = 0.5\frac{f_{ck}}{f_y}\left(1-\sqrt{1-\frac{4.6M_u}{f_{ck}Bd^2}}\right)Bd$$

$$A_{st} = 0.5\frac{20}{415}\left(1-\sqrt{1-\frac{4.6X92.95X10^6}{20X1500X420^2}}\right)1500X420$$

$$A_{st} = 623.18mm^2$$

$$A_{st_{\min}} = 0.0012\times BD$$

$$A_{st_{\min}} = 0.0012\times 1500\times 480 = 864mm^2$$

$$Use, A_{st_{\min}} = 864mm^2$$

Provide 10Ø @ 100 c/c in each direction at bottom of footing i.e. 12 nos .

Step 7: Check for Shear-

The critical; section will be at a distance (d/2) from column face.

Shear Force = Stress X Area $\quad here, Area = [B^2 - (b+d)^2]$

= 293.33X{ 1.5^2-[(0.200+0.420) X (0.300+0.420)] }

= 529.05 KN

Shear stress
$$\tau_v = \frac{V}{b_0 d}$$

$$\tau_v = \frac{529.05}{1 \times 0.420}$$

$$\tau_v = 1260 KN / m^2$$

$$\tau_v = 0.00126 N / mm^2$$

$$here, b_0 \rightarrow perimetre = 2(l+b) = 2(0.2+0.3) = 1m$$

Permisible shear stress

$$= 0.25\sqrt{f_{ck}}$$

$$= 0.25\sqrt{20}$$

$$= 1.11 > \tau_c$$

OK SAFE.

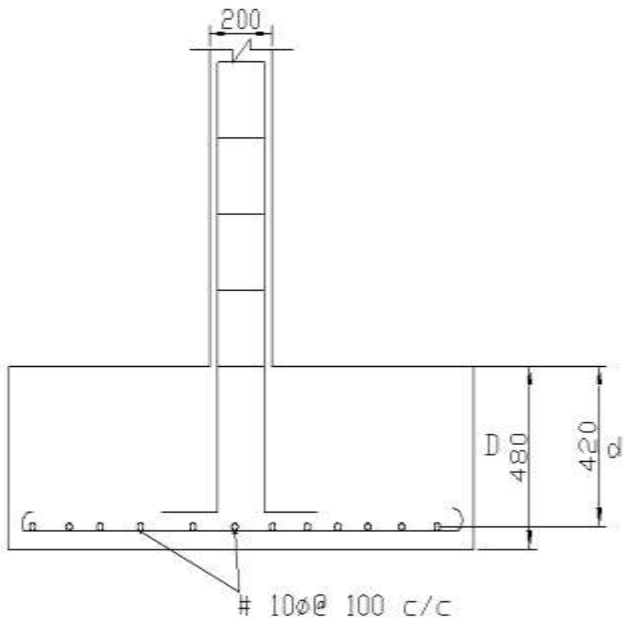


Fig. Sectional View

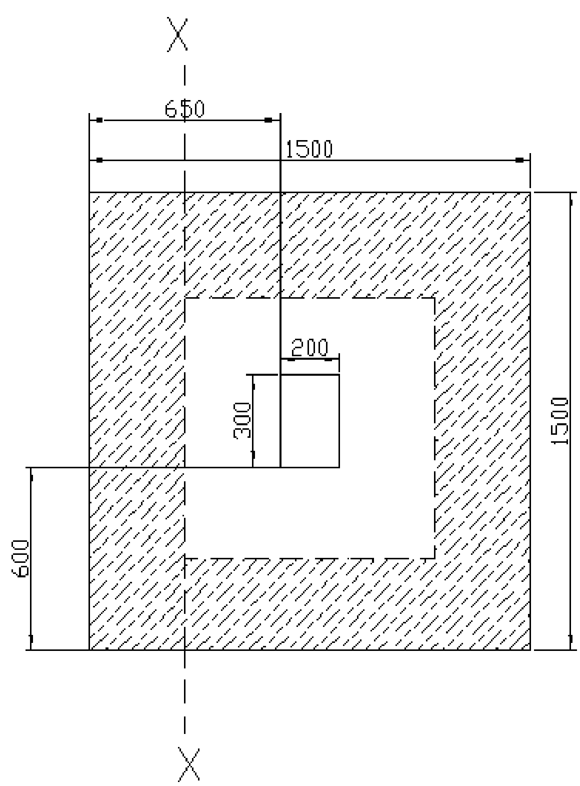


Fig. Plan

CONCLUSION

In this report, a design of Multistory building for residential purpose is presented. We have successfully completed the planning and designing of a multistory (G+2) structure.

The main key features of project are as follows:

- Plot size = 20m X 20m
- Total construction area = 65% of plot size.
- Total no. of 1BHK Flats = 12

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